S
388.1
H3PI
1974

FINAL REPORT

PLANNING INFORMATION STORAGE AND RETRIEVAL SYSTEM

Project Period:

10/11/72 - 4/1/74

Department of Civil Engineering & Engineering Mechanics Montana State University Bozeman

Montana State Library
3 0864 1006 2729 1

FINAL REPORT

PLANNING INFORMATION STORAGE AND RETRIEVAL SYSTEM

Project Period:

10/11/72 - 4/1/74

Prepared for the

STATE OF MONTANA
DEPARTMENT OF HIGHWAYS
PLANNING AND RESEARCH BUREAU

In cooperation with the

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

The opinions, findings and conclusions expressed in this publication are those of the authors and not necessarily those of the Montana Department of Highways or the Federal Highway Administration

Prepared by

Larry J. Coats, Glen L. Martin, Scott H. Danforth, and Michael J. Meldahl

DEPARTMENT OF CIVIL ENGINEERING AND ENGINEERING MECHANICS
MONTANA STATE UNIVERSITY
Bozeman, Montana 59715

March, 1974

TABLE OF CONTENTS

Page

PLANNING INFORMATION STORAGE AND RETRIEVAL SYSTEM
1974 FINAL REPORT 1

Introduction 1
Geometrics 2
Local System Mileage 2
Supervisory Software 3
Roadlog Subsystem 3
True Mileage Subsystem 4
Traffic Subsystem 4
Accident Subsystem 4
Sufficiency Subsystem 5
Railroad Crossing Inventory 5
Bridge Inventory 5
Computer-Based Structural Analysis of Bridges 6
Test Structure 7
Failure Criteria 9
Yield Line Mechanisms 11
Ultimate Capacity of Flat Plate Bridge 13
Load Displacement Behavior 15
Behavior of Other Bridge Types 19
Recommendations 21

REFERENCES 23

LIST OF FIGURES

Figure		Page
1	Drawing of the selected "trial" Montana Department of Highways bridge	8
2	Yield mechanism	14
3	Typical load - deformation curves	16
4	Deflection of positive hinge	20

PLANNING INFORMATION STORAGE AND RETRIEVAL SYSTEM

1974 FINAL REPORT

Introduction

Under this agreement (extending from October 11, 1972, through March 31, 1974) with the Montana Department of Highways, Planning and Research Bureau, priority items were the addition of the following items to the Highway Information System:

1. Bridge Inventory File,
2. Railroad Crossing Inventory File, and
3. Local System Mileage to Roadlog File.

Also, further consideration was to be given to the development of a geometrics file.

Addition of local system mileage to the roadlog file necessitated many changes throughout the Highway Information System. Most access to the files are through the route system, route number, and milepoint. Adding the local system mileage required changing the route number from a 3-digit field to a 5-digit field, and caused major or minor changes to virtually every program in HIS.

The updated Highway Information System is documented in the three volumes Highway Information System Release 3.0: User's Manual, Highway Information System Release 3.0: Programming Details, and Highway Information System Release 3.0: Program Listings. It should be pointed

out that a portion of the work documented in these three volumes was sponsored by the Department of Intergovernmental Relations, Highway Traffic Safety Division. If the separately sponsored portions were documented under separate cover, user confusion would result. The Foreward to each of the volumes indicates the portions of the work sponsored by the respective agencies.

Geometrics

During the time spent on this agreement, little or no interest in developing a geometrics data base was noted in the Department of Highways. Because of this, and with mutual agreement, almost no effort was spent on geometrics.

It is our opinion that the development of a geometrics data base and accompanying software could greatly aid Montana. A working geometrics system could become a parent location system not only for the Department of Highways, but for those agencies involved in such things as land-use information, demographic information, and other societal and environmental information.

Local System Mileage

The addition of local system mileage to the roadlog file involved changing the route number field from three digits to five. For compatibility, this change necessitated changes in file formats in the true

mileage, traffic, accident, and sufficiency subsystems of HIS. A description of the major items performed in each subsystem follows.

Supervisory Software

Because most of the programs in the Highway Information System required updating under this agreement and because the local system addition required a few modifications to the supervisor, this time was chosen to perform a major revision of the supervisor. Several output formatting functions (such as the ability to obtain more than one copy of printed output) were added for the benefit of the user. The interface between the HIS routines and the supervisory software was altered somewhat to increase programming ease and to increase execution efficiency. The system was also made much more flexible for future changes, such as eventual on-line support.

Roadlog Subsystem

The roadlog file format changed considerably when local system mileage was added. Several new fields were added to the record. All programs in the roadlog subsystem required some modification, and a few were completely rewritten.

True Mileage Subsystem

The true mileage file was altered to store 5-digit route numbers and to store an effective date (the date after which the information is valid). All programs in the true mileage subsystem were somewhat modified, and a few were rewritten.

Traffic Subsystem

The traffic file was modified to utilize 5-digit route numbers, and an effective date field was added. All programs required modification, and a few were rewritten.

Accident Subsystem

The accident detail and vehicle files required no modification under this agreement. Accidents occurring on federal aid routes are still reported and stored using only a 3-digit route number. Accidents occurring on local roads are not reported by route number, but by range, township, and section. Programs using only these files required only minor alterations to interface with the new supervisor.

The accident directory file is still stored using a 3-digit route number, although many programs use a re-formatted version of these records with 5-digit route numbers. The accident report and limits files are stored with 5-digit route numbers. Programs

accessing these files required more extensive modifications than those using only the detail and vehicle files, and several programs were rewritten.

Sufficiency Subsystem

The sufficiency file was modified to utilize 5-digit route numbers. All programs needed some modifications, and most were completely rewritten to provide greater system efficiency and make some of the system intricacies more transparent to the user.

Railroad Crossing Inventory

The railroad crossing inventory was added as a new subsystem of the Highway Information System. A file format was agreed on, and some test data loaded. Software was written and tested for maintaining the file and for producing reports from the file.

Bridge Inventory

The bridge inventory was also added as a subsystem of HIS. An agreed upon file format was laid out, and all of the basic bridge information was loaded into the file. Software was written and tested for file maintenance and for report production.

There is an expanding use of computerized and automated data files for many aspects of highway informational systems. It has become necessary to predict possible long-range uses of the stored data so that the present input will be compatible with and allow the greatest future use of this information.

When projecting the possible uses of highway bridge structural data, present day needs must be provided for in such a manner that it may be combined with additional future input and allow a wide scope of now unimagined uses. One of the more obvious use possibilities would be to determine the limiting strength or capacity of all of the state's highway structures. This would be an evaluation of the entire composite structures rather than just the ultimate capacities of the individual elements of the structure. Rowe (1962)* has stated:

> Ultimate load calculations .... are of little use when considering the actual loads bridges are subjected to and become meaningless when abnormal loading is considered since the distribution properties beyond the elastic range must be taken into account. Further, the apparent load factors, deduced for an individual beam, may seriously underestimate the actual load factors for the bridge as a whole. This may be of importance when assessing whether exceptionally heavy loads should be permitted to cross a bridge.
>
> The logical approach to bridge design for heavy abnormal loads, which are infrequently occurring, would be to design the bridge with a specific load factor for this loading and to insure that under normal loading, or working load conditions, the deflection, crack widths, steel and concrete stresses were satisfactory when compared with the permissible values. Although it is not possible, at this stage, to design on the above basis, nevertheless, it is necessary to be able to assess the ultimate load of a bridge structure reasonably accurately for the following reasons:

(i) to give the true safety of the bridge under known or specific abnormal loads,

(ii) to enable safe designs to be prepared for bridges where no rigorous, easily applied design procedure is now available, i.e., skew slab bridges,

(iii) to assist in an understanding of the behavior of the bridge under working load conditions and within the elastic range; this may be of great use in elminating overdesign due to conservative assumptions, and

(iv) to improve the economy of bridge design through an improved appreciation of the behavior of bridges under loads.

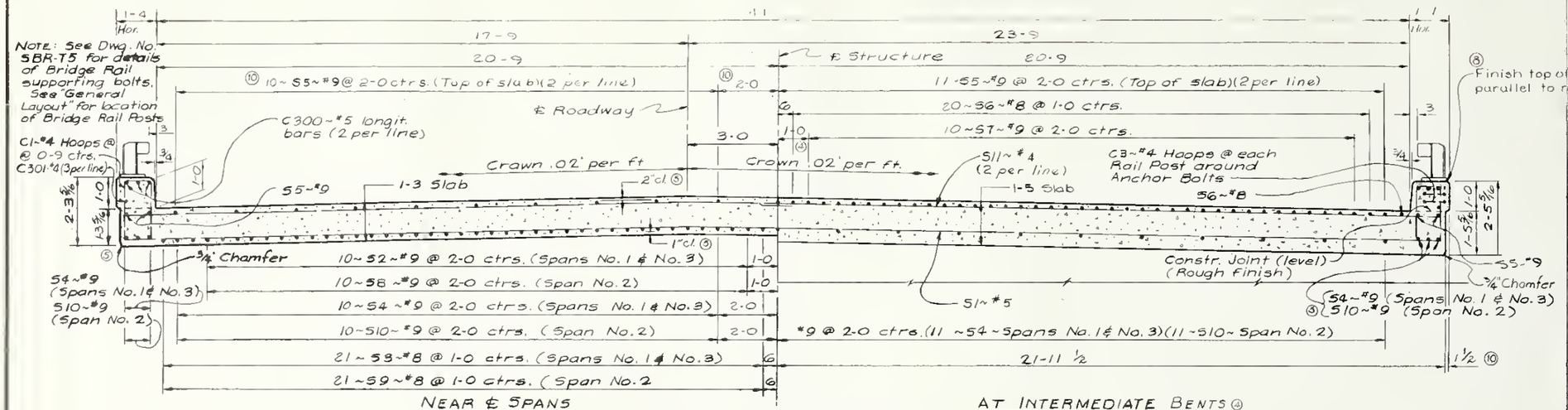
Additional uses might be: defense routing, absolute limiting load on the structures, modification of the limiting structural elements for greater capacity, and determination of the loading causing the first onset of permanent deformation. To test the possibilities and problems associated with the determination of the ultimate capacities and to approximate what units of stored information would be required, one of the more simple types of Montana highway bridges was studied. An attempt was made to predict the possible ultimate load, the likely failure mechanisms associated with this load, and the load-deformation response.

Test Structure

The trial run structure was the Montana Department of Highways standard three span flat slab bridge shown in Figure 1 (Drawing SL-2). The slab is 41.5 feet wide and 15 inches thick in the center span regions and 17 inches thick at the supports. The shallow haunch is 4 feet 10 inches long in the end spans and 6 feet long in

FED. ROAD DIV. NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
8	MONT.			

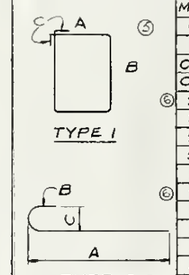
Note: See Dwg. No. SBR-T5 for details of Bridge Rail supporting bolts. See "General Layout" for location of Bridge Rail Posts


17-9
20-9
℄ Roadway
23-9
℄ Structure
20-9
⑩ 10~S5~#9 @ 2-0 ctrs. (Top of slab)(2 per line)
11-S5~#9 @ 2-0 ctrs. (Top of slab)(2per line)
20~S6~#8 @ 1-0 ctrs.
10~S7~#9 @ 2-0 ctrs.
⑧ Finish top of curb parallel to roadway grade
C1~#4 Hoops @ @ 0-9 ctrs. C301~#4 (3per line)
C300~#5 longit. bars (2 per line)
Crown .02' per ft
Crown .02' per ft.
S11~#4 (2 per line)
C3~#4 Hoops @ each Rail Post around Anchor Bolts
S6~#8
1-3 Slab
1-5 Slab
55~#9
2"cl. ⑤
1"cl. ③
¾" Chamfer
10~S2~#9 @ 2-0 ctrs. (Spans No. 1 & No. 3)
10~S8~#9 @ 2-0 ctrs. (Span No. 2)
10~S4~#9 @ 2-0 ctrs. (Spans No. 1 & No. 3)
10~S10~#9 @ 2-0 ctrs. (Span No. 2)
21~S3~#8 @ 1-0 ctrs. (Spans No. 1 & No. 3)
21~S9~#8 @ 1-0 ctrs. (Span No. 2
54~#9 (Spans No. 1 & No. 3)
S10~#9 (Span No. 2)
Constr. Joint (level) (Rough Finish)
S1~#5
#9 @ 2-0 ctrs. (11~S4~Spans No. 1 & No. 3)(11~S10~Span No. 2)
21-11½
54~#9 (Spans No. 1 & No. 3)
③ S10~#9 (Span No. 2)
¾" Chamfer

NEAR ℄ SPANS — AT INTERMEDIATE BENTS ④

BILL OF REINFORCING STEEL (For one structure only)

Note: All dimensions of bent bars are out to out

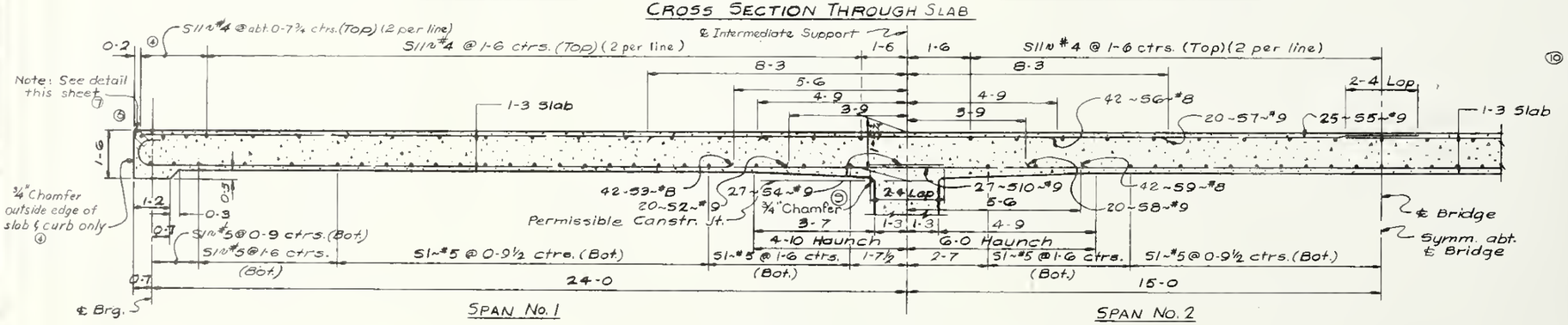


MARK	SIZE	NO.	TYPE	LENGTH	A	B	C
C1	#4	214	1	6'-3"	0'-10½"	1'-11½"	0'-3½"
C3	4	40	1	3-7	0-10	0-8	0-3½
C300	5	12	Str.	40-2			
C301	4	6	Str.	27-0			
S1	5	75	Str.	43-7			
S2	9	40	2	21-11	20-8	1-3	0-10½
S3	8	84	2	20-0	18-11	1-1	0-9
S4	9	54	2	26-10	25-7	1-3	0-10½
S5	9	50	Str.	40-8			
S6	8	84	Str.	9-6			
S7	9	40	Str.	16-6			
S8	9	20	Str.	22-6			
S9	8	42	Str.	19-0			
S10	9	27	Str.	32-4			
S11	#4	110	Str.	22-4			

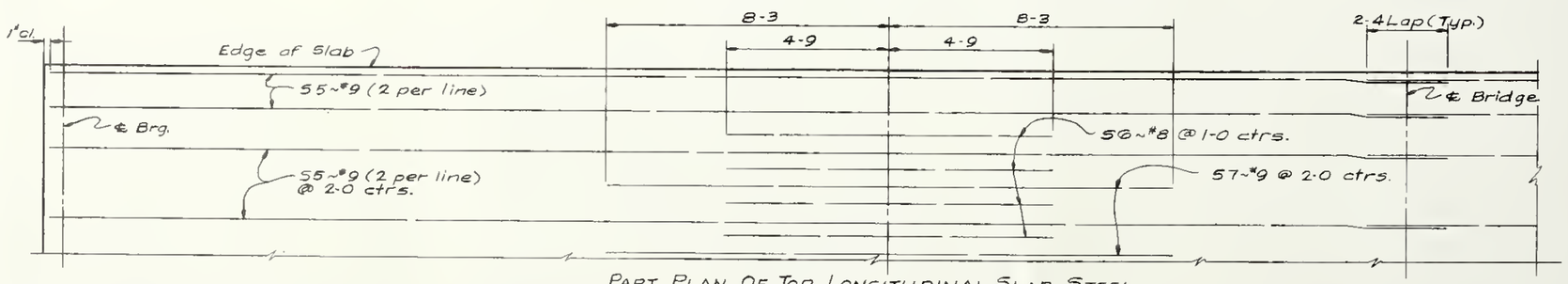
ESTIMATED PLAN QUANTITIES

⑧	CLASS "AD" CONCRETE	173.7 Cubic Yards
⑩⑥⑤④	REINFORCING STEEL	36,982 Pounds
⑦	Structural Steel	628 Pounds

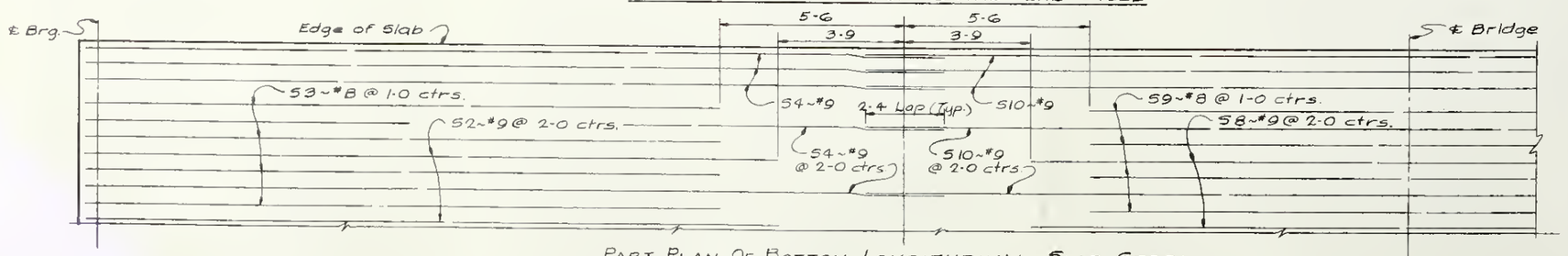
CROSS SECTION THROUGH SLAB



HALF LONGITUDINAL SECTION THROUGH SLAB ④



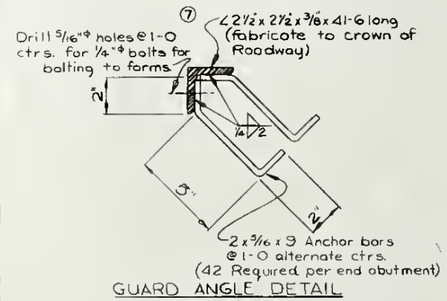
PART PLAN OF TOP LONGITUDINAL SLAB STEEL



PART PLAN OF BOTTOM LONGITUDINAL SLAB STEEL

⑨ Note: Guard Angle may be furnished in two pieces with end anchor bars 6" maximum from each end. The Guard Angle, if furnished in two pieces, shall be field welded and held in accurate position while concrete is placed.

⑧ Note: Paint all guard angle surfaces not in contact with concrete with one (1) shop coat and two (2) field coats of paint as provided in the Standard Specifcations


⑦ L2½"x2½"x⅜"x41-6 long (fabricate to crown of Roadway)
Drill 5/16"ø holes @ 1-0 ctrs. for ¼"ø bolts for bolting to forms
2"
2"
2½"
3"
2"
2 x 3/16 x 9 Anchor bars @ 1-0 alternate ctrs. (42 Required per end obutment)

GUARD ANGLE DETAIL
SCALE: 3"=1'-0"

⑩	REVISED	R.N.D.	12-29-72
⑨	REVISED	R.N.D.	7-31-72
⑧	REVISED	R.N.D.	3-31-72
⑦	REVISED	N.H.R.	3-29-71
⑥	REVISED	R.N.D.	12-31-70
⑤	REVISED	J.C.H.	12-31-69
④	REVISED	R.N.D.	7-25-69
	REVISED	R.N.D.	7-1-69
	DESIGNED	L.K.L.	3-15-66
	DRAWN	F.L.S.	3-23-66
	TRACED		
	CHECKED	K.G.R.	4-21-66
	REVISED	K.G.Kapp	11-14-67
	REVISED	R.N.D.	12-19-68

STATE OF MONTANA
DEPARTMENT OF HIGHWAYS

STANDARD FLAT SLAB ℄ CURB DETAILS
FOR THREE SPAN CONTINUOUS
24'-30'-24' SPANS - 41-6 FT. RDWY.
HS 20-44 OR ALTERNATE LOADING
0-3 CURB ℄ STD. STEEL BRIDGE RAIL
TYPE N.D. 5
Scale ½"=1'-0" Except As Noted

APPROVED BY _____ BRIDGE ENGINEER

DRAWING NO. SL-2

the interior spans. Top and bottom steel varies as shown. The curb beams extend the entire length of the structure and are 16 inches wide and 27 to 29 inches deep. A simple support exists at the abutments. The continuous structure is supported by three 24-inch diameter columns and an integrally built cap beam at two interior locations.

This deceptively simple appearing structure in no way represents a "typical" Montana Highway Department bridge. The bridges of the State contain a myriad of material usages and structural configurations. Yet, most of the problems associated with ultimate capacity concepts are clearly illustrated by the less complex structural arrangement. Anisotropic steel configurations present complications similar to the more obvious geometric variations of other structures. No single structure could cover all variations and time did not allow the cataloging of several bridge arrangements.

Failure Criteria

The theoretical capacity of a particular reinforced concrete section depends only on the quality of the steel and concrete and upon the volumes used and the particular geometric arrangements.

Just as judgments were made in establishing realistic allowable design stress values in the working stress methods and the material and performance reductions and load factor increases in ultimate strength methods, so too, when attempting to evaluate the

absolute maximum capacity of a structure, rational limitations and interpretations must be made. In the end, the absolute maximum capacity whether it be a load or a deflection limitation is a judgment and must remain so short of actual destruction loading of the bridge.

The first judgment to be made is whether or not to include the customary usage and workmanship reduction factors. Assuming a valid probabalistics basis exists for this reduction, it reasonably should be applied.

Next, consideration of the imaginary failure load leads to the assumptions of some vehicular wheel pattern that will cause the proof loading. Much greater total load can be carried by a pattern of loads as opposed to a single concentrated load.

Given the theoretical strength capacities at all the various geometric changes in the bridge and a possible wheel configuration, the most probable mode of flexural failure must be established. Flexural failure is designated because through the process of yielding the steel, moment redistribution, and hinging, larger loads may be carried and a long period of ductile deformation will preceed collapse. Other modes of failure are possible and most will be brittle non-warning modes. These include diagonal tension, bond, torsion, and structural stability in the main members and failure of supporting details such as seats and rockers and brackets. All of these must be investigated to assure that no premature failure will prevent the development of the desired mechanism.

The actual "mechanism" of collapse is probably best represented by various arrangements of ductile beam hinges and slab yield lines. The most probable of all rotational arrangements will be that one which requires the least internal energy for formation or the least load for static equilibrium of both internal and external forces and moments.

The formation of the planes and lines of rotation require that all sections contain less than the balanced amount of reinforcement. This decreases the stiffness such that ductile hinging and moment redistribution are possible. As the load is increased to produce the ultimate moment, more and more micro cracking and bending curvature develops until a beam hinge or a slab yield line forms. At the yield of the steel the rotations become "plastic" and the ultimate moment on that particular section is reached. As more load is applied, rotation of a considerable magnitude takes place under this constant plastic moment. This rotation continues while at other locations other hinges and lines are forming. All yielding locations are assumed to be joined together by straight undeformed portions of the structure. As the lines spread from points of high local moments to the boundary of the unit, the planar sections thus formed will eventually become a selfdeforming mechanism that will proceed to collapse without additional input. The main difficulty is to establish the most probable mechanism pattern.

A good deal of experimental work on models and full-scale structures substantiates the validity of the yield line concept -- Rowe (1962), Nagaraja and Lash (1970), European Committee for Concrete (1962), and ACI Publication SP-26(1969). Many properties and characteristics of yield lines have been formulated to assist in a determination of the most probable pattern of line development -- European Committee for Concrete (1962), and Jones and Wood (1967). The process is tedious. All possibilities must be checked and they depend upon the overall geometry, local changes in geometry and sectional properties, the load configuration, the moment sense at various locations, and the direction and relative contribution of the steel.

Once the least work mechanism has been established, the usual procedure in design would be to apply the desired overload margin and then proceed with the slab design. We are then concerned with a structure that already exists and the next logical step is to establish a reasonable fraction of the collapse load as the absolute maximum allowable load ever to be placed on the structure. In experimental studies a normal variation between calculated and test loadings is approximately $\pm$ 10%. The Department of Highways may at this point set a load limit with full knowledge of the total capacity of the structure. This is not the case at the present time where bridges are rated as some fraction of a standard load classification which conveys no measure of the total structural strength. Also, once the maximum load configuration and magnitude is determined and reduced to a maximum permissible load, then all

lesser loads would be acceptable and separate computations for all unusual loadings no longer would be required.

Ultimate Capacity of Flat Plate Bridge

The ultimate moment capacity was computed for all changes in section and all steel arrangements. These moments were assumed to be the plastic hinge moments.

Using the least work methods, and evaluation of the more rational yield patterns established the most likely collapse mechanism to be two straight transverse lines across the end span slabs and curb beams. One line, the first to form, occurs at the point of maximum positive moment and the other forms over the first interior supports at the point of maximum negative moment.

The load configuration and magnitude assumed to result in the foregoing moments and mechanism was two axles four feet apart with each axle carrying 363 kips. It was assumed that the ultimate vehicle would be the only one on the structure and moving so slowly that it would approach a static load situation. Other load systems would yield different ultimate load values. Perhaps the Department of Highways would find it useful to attempt to extrapolate present day loadings and loading trends to estimate a more probable future maximum vehicle. This would then result in a different maximum load value.

The results of the present trial run are summarized in Figure 2.

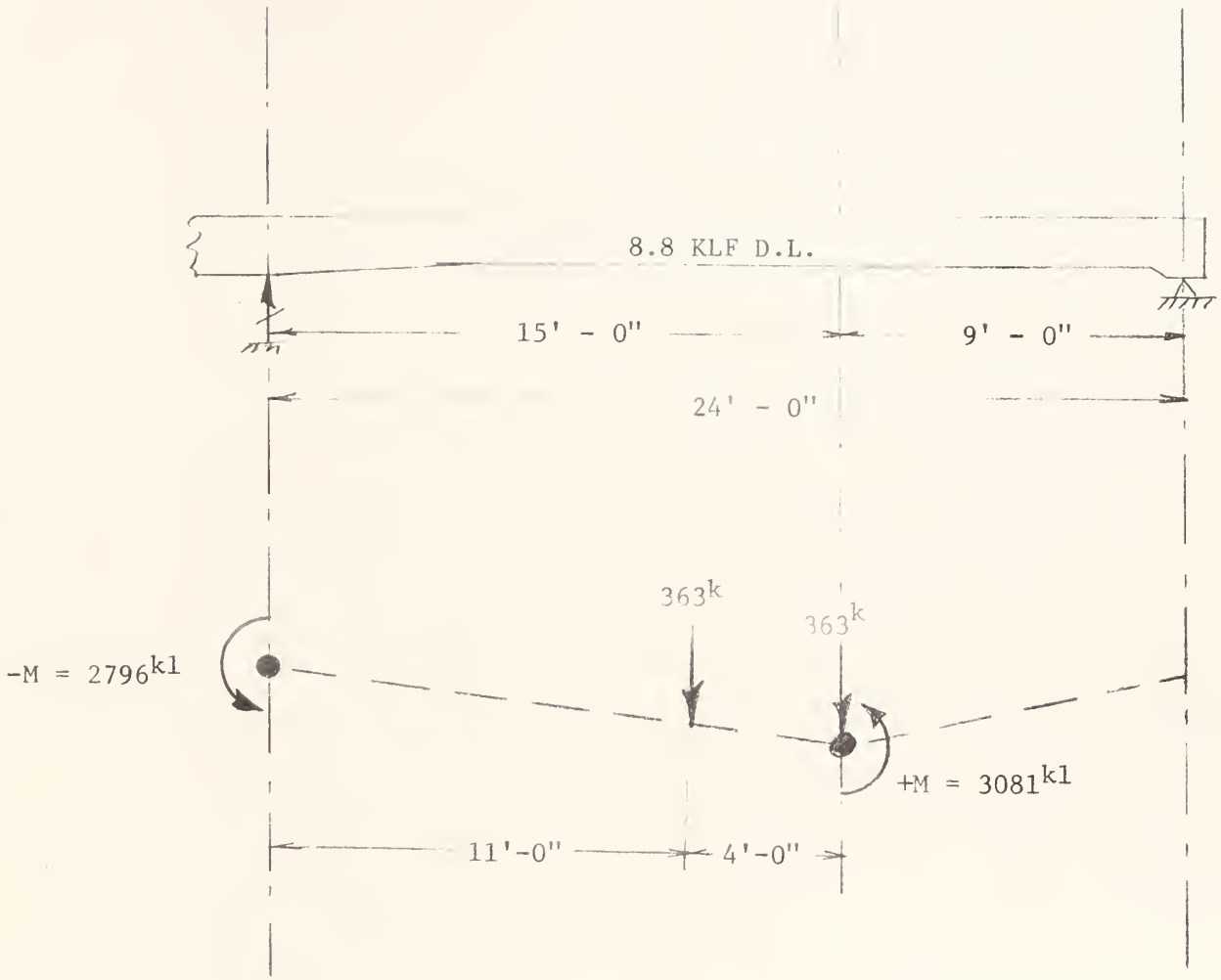


FIGURE 2. Yield mechanism.

The load displacement relationship is essential if the behavior of a structure is to be examined for loads greater than the basic design loads. If a complete description of the load deformation curve is available, the loads at which nonlinear behavior begins can be determined and a more realistic estimate of the safety of the structure can be made than if only a linear analysis is carried out.

The load displacement response of structures depends on several factors including the material, the fabrication details, and the structural geometry. Generally, structures are fairly ductile because there is a good deal of inelastic deformation prior to complete failure. This ductility depends on the degree of redundancy of the structure to a certain extent because in a highly redundant structure a lot of redistribution of load takes place as the individual members or parts of the structure become inelastic. This is illustrated by curve A, Figure 3. Simple structures exhibit ductility, however, the overload capacity beyond first notice of nonlinear behavior is generally not as great for simple structures as it is for highly redundant structures. The behavior of a simple structure may be somewhat like that shown by curve B, Figure 3. As can be seen, collapse of a structure of this type occurs at a load only slightly greater than the load causing onset of nonlinear behavior. Curve C in Figure 3 illustrates a third type of behavior that can be exhibited. This illustrates a rather brittle or low ductility failure in which there is very little deformation before

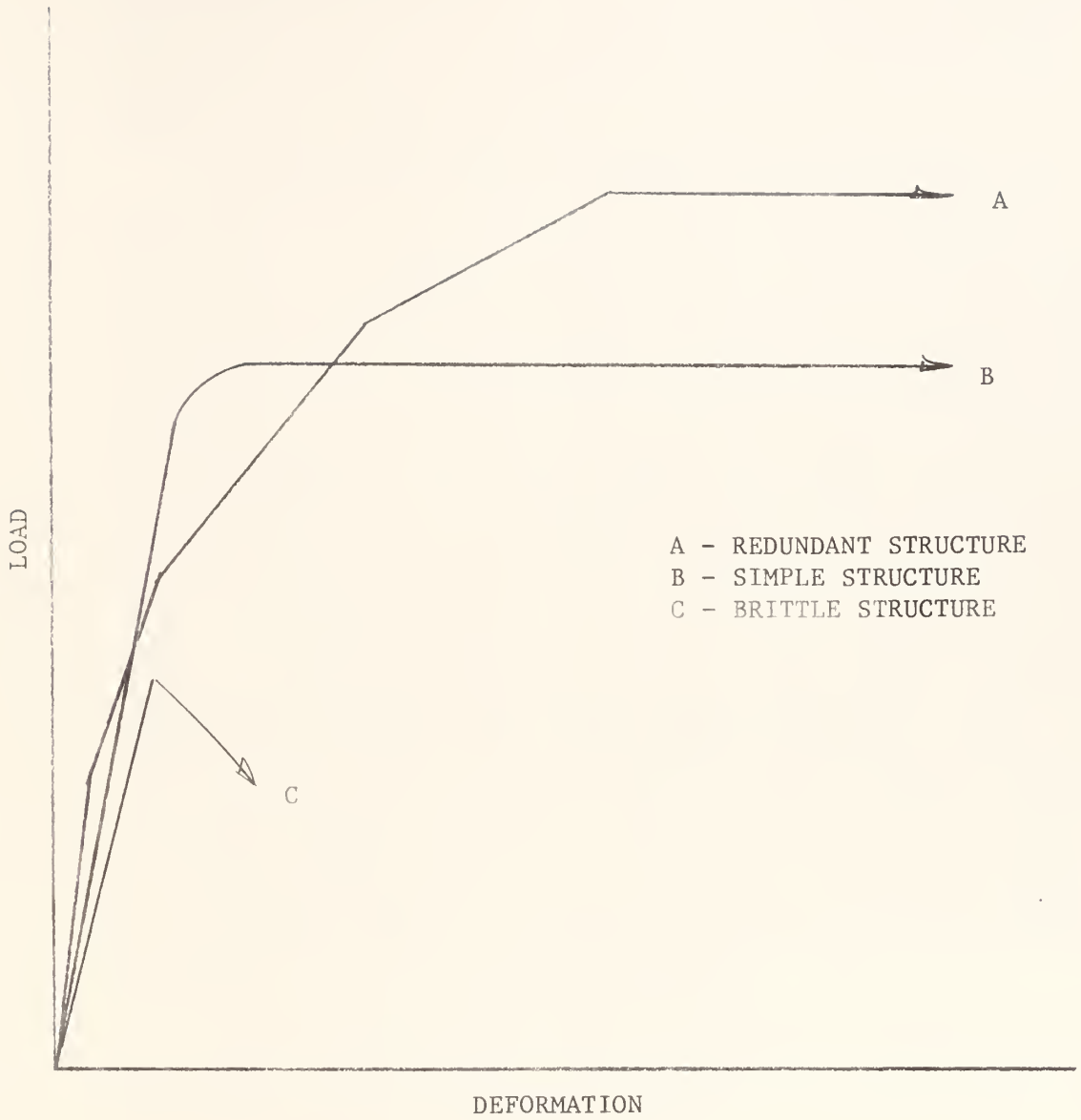


FIGURE 3. Typical load - deformation curves.

the structure breaks up. This kind of a load deformation curve can be caused by improper use of brittle materials in the structure, by bad design, bad fabrication, or by various types of instabilities. Modern structures, if properly designed and fabricated, rarely exhibit this brittle behavior.

As shown by the curves of Figure 3, the true safety of a structure is not well estimated if only an elastic response is available. For example, if design were done elastically for both structures A and B, using a factor of safety with respect to first yielding of 3, the actual factor of safety of each structure would be quite different. As can be seen for the simple structure, the actual factor of safety is probably quite close to two. However, for the redundant structure, the actual factor of safety is much greater than two with respect to failure even though it would be approximately equal to two at the onset of nonlinear action. It may be possible, therefore, to allow an overload once or twice for the highly redundant structure even though this overload caused some inelastic behavior. On the other hand, allowing overloads for the simple structure may not be advisable.

A load deformation curve was obtained for the test structure. The actual dead load was used and the live load was allowed to vary from zero to the ultimate value of 363 kips. The live load configuration was the AASHO alternate loading, described in Standard Specifications for Highway Bridges (1969), which consists of two axle loads spaced four feet apart. Deflections were computed for dead load acting alone, for the dead load plus live load causing the first yield line to form, and for the case just prior to

collapse. The deflection computations were done using the ACI formulas, as described in Building Code Requirements for Reinforced Concrete (ACI 318-71) (1971), for the modulus of elasticity of the concrete and the moments of inertia of the concrete cross-sections. Using the ACI formulas gives a good estimate of the deflection for low values of load. As the loads approach ultimate, however, the ACI formulas underestimate the deflections since they do not account for large localized curvatures in regions of high moment. Therefore, the deflections at the high loads are probably underestimated 10 to 20 percent. More accurate deflections could have been obtained by developing complete moment-curvature relationships for the structure at high loads. This, however, was beyond the scope of this study.

The load deflection results are shown on Figure 4. As can be seen, the deflection increases rapidly after formation of the first yield line. When the second yield line forms complete collapse of the structure occurs. Also plotted on Figure 4 are the points for the AASHO design Group I loading which consists of the dead load plus live load plus impact. In obtaining this loading it was assumed that three lanes were loaded and a reduction factor of 0.9 was applied to the total of the three loads. Also shown on the plot are the deflections for the AASHO factored load which consists of 1.3 times the dead load plus 2.17 times the live load plus impact. This point does not plot on the curve because the curve was obtained using a constant dead load rather than the dead load with the factor 1.3 applied to it. As can be seen, even with the factored load applied, there is a good deal of reserve strength remaining in the structure.

After the second yield line forms the structure becomes a mechanism and addition deformations can take place with no additional increase in load. For the reinforced concrete slab section considered in this example the amount of hinge rotation should be quite large and an additional large deflection could be expected from this bridge prior to its complete breakup. This is, therefore, a good example of a ductile type failure. The failure has been assumed to be a flexural failure which requires adequate shear and bond resistance in order to be achieved. Both the shear and bond were checked in this case and they appear to be adequate. Thus the bridge would be expected to act in the manner indicated by the curve of Figure 4.

Behavior of Other Bridge Types

The test problem illustrated in this report was for one type of bridge used by the Montana Department of Highways. There are many other types of bridges which are in common use and there are many others that are unique. In general, given the geometric and material properties for the bridges it would be possible to perform an analysis similar to that performed for the test bridge. The failure mode would have to be determined and in some bridges this might pose a more serious problem than determining the failure mode for the slab bridge investigated here. The problem is soluble, however, and methods for solution if not available now certainly will be available in the future. Basically, the steel bridges and

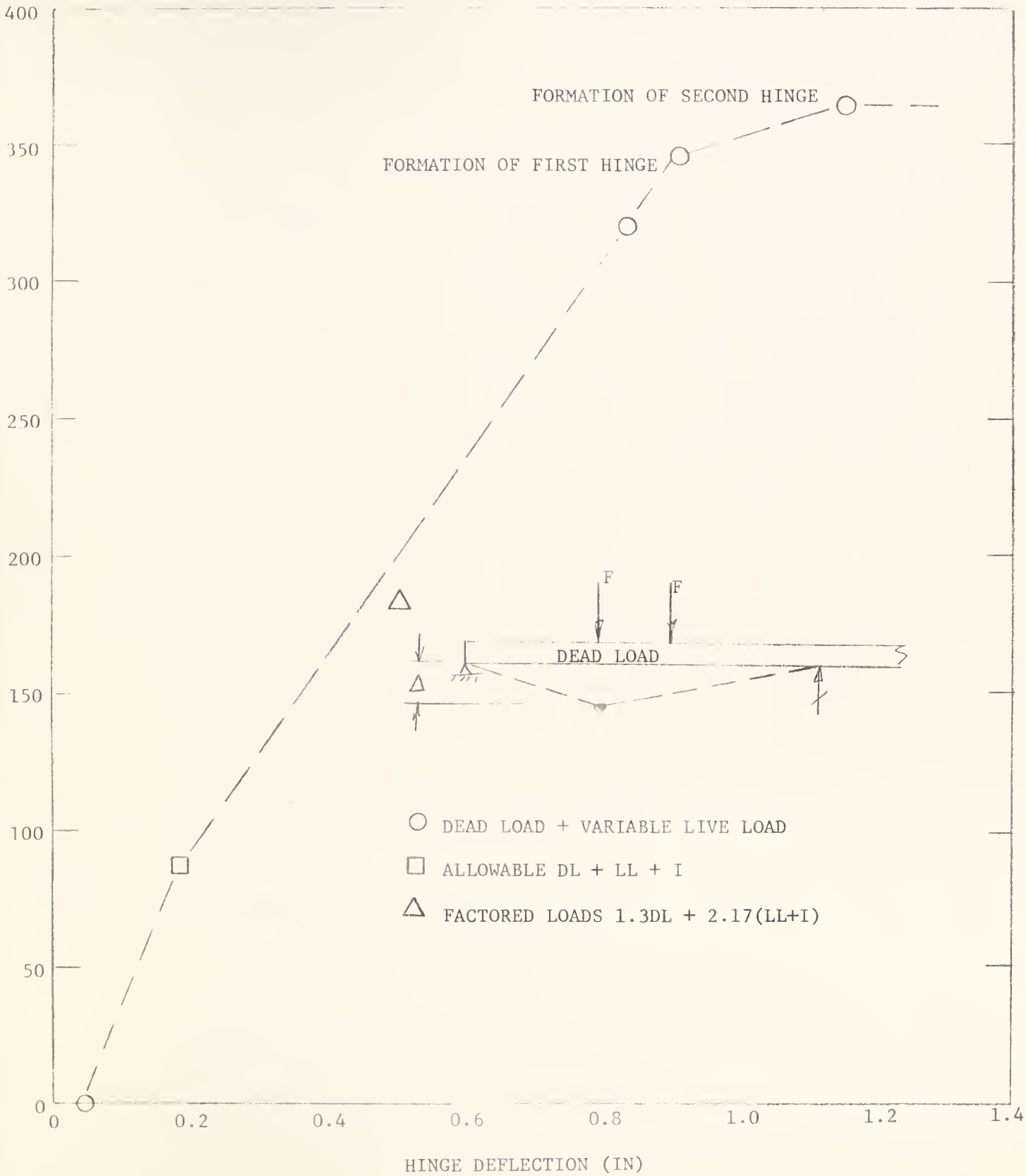


FIGURE 4. Deflection of positive hinge.

the concrete bridges will behave in a similar fashion. In the case of steel it may be possible to have a rather brittle failure should the stiffening be inadequate and instability be able to occur at a low value of load. Brittle behavior in steel also could occur if a brittle fracture of the material takes place. By and large, however, steel bridges will fail in a ductile manner. The procedure used for determining the failure of the bridges will be a nonlinear analysis similar to the procedure used for the concrete bridge. If the bridge is a truss bridge the procedure would be to determine the incremental collapse of this bridge as each member reaches its limiting capacity. If the truss is a simple truss, the total load that can be carried will be determined by the capacity of the weakest member. On the other hand, if the truss is highly redundant, there will be a great deal of redistribution of stress after the most highly stressed member yields allowing the load to increase considerably beyond the load that occurs at first yielding. A truss bridge, of course, also can fail through a compression buckling of the members and the instability would have to be considered in developing an ultimate strength model.

Recommendations

The study such as has been performed here would be a valuable addition to the store of information available to those involved with the design and maintenance of Montana's bridges. A bridge file which is being designed for future uses should, therefore,

contain the information necessary to perform such an analysis. This means that for each bridge data would have to be available giving the complete geometric and material properties for the bridges. If this data were available in a format compatible with existing structural analysis and computer programs, then as the state-of-the-art advances, up to date detailed studies could be performed for the individual bridges. This would require a minimum of input since the input data would already be available on the bridge file. The degree of automation for computing collapse of structures and for obtaining complete deformation curves for any type of specified load configuration is a matter of conjecture at this time. Due to the practically infinite variety of yield lines that can form in a structure it may not be feasible at the present time to completely automate a yield line analysis for a complex structure. However, should this kind of analysis become automated, and if the bridge data were available, it would be a simple matter to take the available data and run it on the automated program. For more immediate use, should the bridge engineer wish to study the behavior of existing structures, he can do so using existing computer programs and the data that would be available on file. For example, right now using STRUDL he can perform an elastic linear analysis on any structure for which the geometric and materials data is available. He can do this for any load configuration that may be important. Also, if the data were on file he could more efficiently do the sort of analysis that was carried out for the test problem. Deflection computations, cross-sectional properties, ultimate moment capacities all could have been computed using STRUDL and the bridge properties on a computer file.

REFERENCES

ACI Publication SP-26, "Second International Symposium Concrete Bridge Design," ACI, 1969, pp 411-552.

Building Code Requirements for Reinforced Concrete (ACI 318-71), American Concrete Institute, 1971.

European Committee for Concrete, "Information Bulletin No. 35," Cement and Concrete Association, London, 1962, English Translation.

Jones, L.L., Wood, R.H., "Yield-Line Analysis of Slabs," American Elsevier Publishing Co., 1967.

Mills, G.M., "The Yield-Line Theory: A Programmed Text," Concrete Publications Ltd., London, 1970.

Nagaraja, R. and Lash, S.D., "Ultimate Load Capacity of Reinforced Concrete Beam-and-Slab Highway Bridges," ACI Journal Proceedings, Vol. 67, No. 12, Dec. 1970, pp 1003-1009.

Rowe, R.E., "Concrete Bridge Design," John Wiley & Sons, Inc., 1962, pp 203-225.

Standard Specifications for Highway Bridges, American Association of State Highway Officials, Tenth Ed., 1969.

Wang, C.K., Salmon, C.G., "Rinforced Concrete Design," International Textbook Co., 2nd Ed., 1973, Chapt. 18.